PRELIMINARY TESTING OF THE DECK CONCRETE OF THE MOUNT HOPE BRIDGE

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RHODE ISLAND DEPARTMENT OF TRANSPORTATION
For the Rhode Island Turnpike and Bridge Authority
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ABSTRACT

The Mount Hope Bridge is a suspension and carries SR 114 over the small inlet between the Mt. Hope Bay and the Narragansett Bay between Bristol and Portsmouth. Due to concerns over the deterioration of the fourteen year old bridge deck, RIDOT was asked perform some preliminary testing to determine possible causes. The deck consists of a weathering steel orthotropic grid, embedded in concrete. The concrete's function is to provide cover for the steel and create a riding surface for traffic. It is apparently not intended to be structural. However, forty percent of the surface has been patched over the last five years, far in excess of what would be expected.

Several tests were performed, within a work zone provided by the repair operation. These tests included: Cores for compressive strengths (ASTM C 39), cores for rapid chloride permeability (AASHTO T-277), half-cell potentials (ASTM C 876), powdered samples for chloride contents (AASHTO T-260), testing for specific gravity (AASHTO T 85), Windsor Probe readings (ASTM C 803) and a delamination survey by chain drag (ASTM D 4580). A visual survey was also conducted of the test area, mapping test locations and relative age and placement of patched areas. Two specimens were also sent out to Construction Technologies Laboratories for petrographic analysis and the report will be forwarded for review when it is received. This data is intended to help confirm the conclusions stated herein and is not essential to the analysis.

The data suggests that chloride induced corrosion, as a combination of highly permeable concrete and the weathering steel grid (*formulated* to oxidize) and damage to the concrete to remain during removal of deteriorated sections have all contributed to the rapid deterioration of the bridge deck. There may also have been mechanically induced cracking due to curing effects and bridge dynamics. At this point, the only viable options appear to be replacement of the wearing surface with a less permeable material and if possible, treatment of the steel to slow the corrosion process.

OBJECTIVE

This preliminary study has the intent of determining the causes of the initial and accelerated deterioration of the Mount Hope Bridge deck and to suggest possible treatments to minimize further distress.

BACKGROUND

The Mount Hope Bridge was constructed in 1930 and serves as a major access through the East Bay area. It is a suspension bridge with a two lane roadway and a narrow shoulder on either side. In 1986, the bridge deck was replaced with an orthotropic steel grid constructed of A 588 weathering steel (an alloy formulated to rust, creating a passive layer to protect the steel). The grid cells are four inches wide (center to center, measured in the transverse direction and parallel to the longitudinal axis of the deck) and nine inches long (center to center, measured in the longitudinal direction). The longitudinal plates are approximately one-quarter inch thick and the transverse plates are approximately five-eighths of an inch thick. Both are three inches tall, with the transverse plates being slightly higher than the longitudinal. The grid is embedded in concrete, with approximately two inches of cover on top and there is a 20 gauge galvanized steel sheet on the bottom, used as a form during the original placement of the concrete (based on discussions with the consultant). The total thickness of the deck is six and a half inches. The consultant has stated that the concrete is not structural. The maximum air content was specified as 9.5%, but was exceeded on occasion. The riding surface has been tined to improve traction. However, extensive patching of the deck has reduced the tined surface area by nearly half.

About seven years ago, after seven years in service, spalled areas began to appear on the deck and the rate of deterioration has increased to the point that forty percent of the surface has been patched to date. Although the patches seem to be holding up well, in many locations there are several generations of repaired areas adjacent to one another. As the spalled areas are sounded prior to removal of the concrete to determine the extent of the delamination, any new spalls in adjacent areas are likely to have initiated after the previous patches have been made.

[NOTE: No plans, specifications or data from the original deck replacement project were available for review prior to the writing of this report.]

TESTING PLAN

Based on the air content, the type of steel (weathering), the distress pattern, the repair method and the dynamics of the bridge movement, several tests were selected to determine possible causes of the distresses. The deck section considered extended south from the north tower 120 feet in the northbound lane.

- DELAMINATION SURVEY: Chain dragging to sound out delaminations was employed to survey the area considered. A pole with a transverse cross-member with attached lengths of chain was dragged across the deck. Normal, solid bonded concrete has a dull sound when chain dragged. When a delamination is found, the pitch of the sound rises (creating a hollow sound), owing to the lesser thickness of the concrete solidly in contact with the chains. In this way, the areas of distress can be mapped out over a wide area.

- CONCRETE CHLORIDE CONTENT: Chloride samples were taken to determine the extent of the chloride intrusion in the deck concrete. Three depths for each location were collected to obtain a profile (intrusion level vs. depth) of the chloride content. Chloride contents above 1.3 lbs/yd³ (in concrete with ¾" nominal aggregate) can cause corrosion of the embedded reinforcing steel. In order to get a picture of the chloride levels in the deck in three dimensions, samples were taken at various points along the length and width of the work area, as well as at varying depths.

Photo 1 – Chloride Sampling

- STEEL CORROSION: Half-cell potentials were taken in a limited area on the deck. The test considers the bridge deck to be a battery, with the steel acting as an electrode and the concrete performing the function of an electrolyte. When active corrosion is occurring, the voltage created by the battery cell typically falls within a certain range (-0.20 to –0.35 volts). The intent is to get map of subsurface corrosion and predict future distresses.

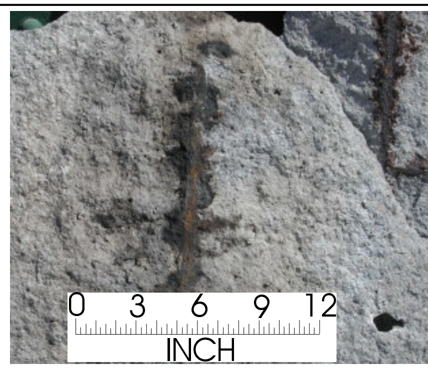
- ELECTROCHEMICAL INDICATION OF CHLORIDE PERMEABILITY: Electrochemical testing to approximate the measure of the concrete permeability indicates how susceptible the concrete may be to chloride intrusion. When high numbers show up in concrete (measured in units of charge or current over time), it will not provide a sufficient barrier to intrusion. These values are used in concert with the chloride contents to assess the potential for problems that may arise from corrosion of the steel reinforcement. Two core sizes were used. One, at four inches in diameter, is the standard for the test. The second, at two and three-quarter inches, was used to correlate the effects of lack of coarse aggregate on the permeability. Additionally, chloride samples were taken in the vicinity of the larger cores. This was done to check the relative initial chloride content in the concrete, which will affect the final permeability result.

- IN-SITU STRENGTH DETERMINATION: Windsor Probe tests were used to gauge the strength throughout the deck. This involves using a calibrated load to fire a metal rod into the concrete. The depth of penetration can then be used to indicate the concrete compressive strength. The aggregate is mostly fine material and interpreting the Windsor data is normally dependent on knowing the Moh's number of the stone in the concrete matrix, but the deck mix is closer to a mortar than a standard concrete. So it was thought that the results might be skewed. Therefore, cores for compressive strengths were taken for correlation purposes.

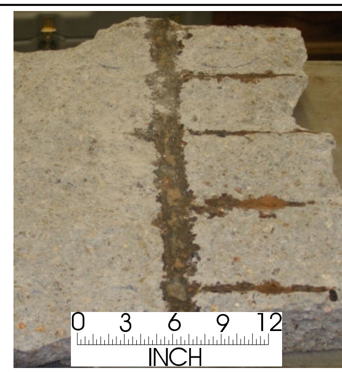
- VISUAL SURVEY: A visual inspection was also performed to gain a subjective perspective of the distresses in the deck and to try to ascertain whether there is any pattern. A map of the work area was made showing the test sites and patches (See Figure 1, p. 11). The samples in the photos below exhibit significant corrosion induced spalling. Note in Photo 3D the dirt on the edge of the sample, indicating a pre-existing crack (prior to concrete removal) directly above the rust stain.

Photo 2 – Windsor Probe Testing

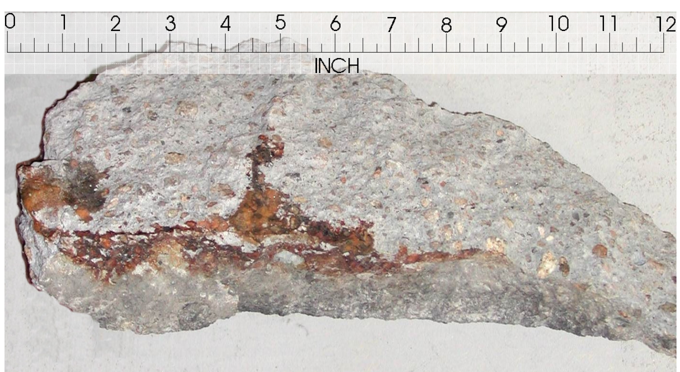
A – Corrosion Spall



B- Corrosion Spall



C – Planar Corrosion Spall



D – Corrosion Under a Vertical Crack

Photo 3 – Corrosion Evidence

In photo 4, the repair operation is shown. The edges of the openings are clean and well formed and the grid is penetrated. Note the use of jackhammers to remove the concrete, which is common in such situations. Note also the fineness of the aggregate in the concrete in these pictures. In Photo 5, note the small size of the aggregate and the large percentage of mortar.

Photo 4 – Patching Operation, Showing Steel Grid

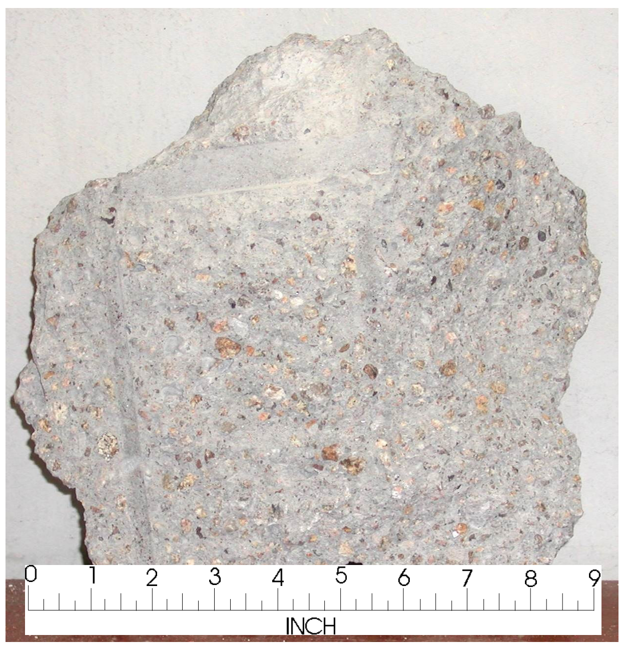


Photo 5 – Close View of Concrete Matrix

TEST DATA

Table 1 - Windsor Probe Data:

Reading Set No.	Longitudinal Reference Pt. (Plate No.)	Curb Offset[1]	Reading (in)	Correlated Strength (psi)
1	34	3’8”	2.000	2900
	34	7’8”	1.975	2800
	34	10’7”	1.975	2800
2	33	2’2”	2.000	2900
	33	6’8”	1.875	2400
	33	9’9”	2.000	2900
3	32	2’	1.925	2600
	32	6’5”	1.925	2600
	32	9’10”	1.925	2600
4[2]	31-2’	2’6”	1.850	2300
	31-2	7’1”	1.950	2700
	31-8’6”	10’10”	1.525	1000[3]
5	30	2’6”	1.925	2600
	30	7’	2.000	2900
	30	10’10”	1.975	2800
6	29	2’7”	1.875	2400
	29	6’8”	1.750	1900
	29	11’8”	1.900	2500
7	29-19’	2’6”	1.425	600[3]
	29-19’	6’7”	1.775	2000
	29-19’	12’	1.875	2400
8	29-13’6”	6’5”	1.875	2400[4]
	29-13’6”	7’	1.875	2400[4]

Note:
[1]Tests were performed primarily in wheel paths and midway between wheel paths.
[2]Reading set 4 offset longitudinally to avoid patched areas.
[3]These results are likely outliers, based on an average of 2409 psi and a standard deviation of 578 psi, including the low values and an average of 2561 psi and a standard deviation of 283 psi, neglecting those values.
[4]Reading sets 7 (center of wheel paths) and 8 adjacent to cores C and D (respectively) for compression testing for correlation purposes. Increments are based on tables supplied with Windsor Probes. Correlation value rounded up.

Table 2 – Compressive Strength

Specimen	Length (in)	Diameter (in)	Load (lbs)	Strength (psi)
A	3.31	1.75	10,300	4280
B	3.31	1.75	9,920	4120
C	3.31	1.75	5,920	2460
D	3.38	1.75	5,760	2390

Note: Specimen B had a 0.25 × 1.25 inch plate oriented parallel to the long axis of the core, 0.38 inches from the top of the core and 0.38 inches from the center. After compressive failure, no signs of distress due to the plate were visible.

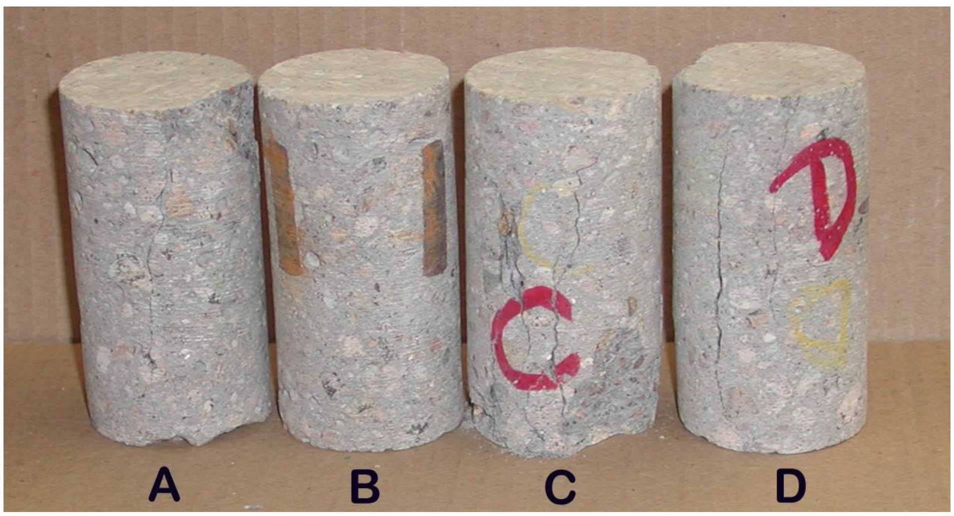


Photo 6 – Patterns of Compressive Breaks

Table 3 - Half-Cell Potentials (Volts), 2.5 ft × 2.5 ft grid spacing:

Grid	W1	W2	W3	W4	W5
L1	.074p	.045p	.025p	.013	.011p
L2	.020	.015	.012p	.008p	-.028p
L3	-.023p	-.038p	-.031	-.036	-.027
L4	-.028p	-.030p	.015	.017	.016
L5	.020	.020	.023	.011	.010
L6	.012	.010p	.011	.019	.008p

Note: "L" designation for longitudinal direction, "W" for transverse. "p" suffix indicates reading taken on a patched area. Values in the range –0.20 to –0.35 volts indicate active corrosion in the steel. Values more negative than the range indicate a 90% probability of active corrosion and values more positive have a 90% probability that the steel is passive. Weathering steel may also be less active electrically than normal steel when corroding.

Table 4 – Chloride Contents:

Field Samples

Hole No.	Specimen Depth (in)	Chloride Content (lbs/yd^3)
1	1	9.24
	2	5.28
	3	1.84
2	1	7.76
	2	5.52
	3	1.68
4	1	2.48
	2	0.68
	3	0.48
5	1	3.52
	2	2.20
	3	0.84
6[1]	1	10.12
	2	5.04
7	1	6.68
	2	3.04
	3	1.00
8[2]	1	5.40
	2	1.40
	3	0.48
9[2]	1	8.20
	2	2.72
	3	0.80

Lab Samples – Collected Adjacent to Four Inch Permeability Cores

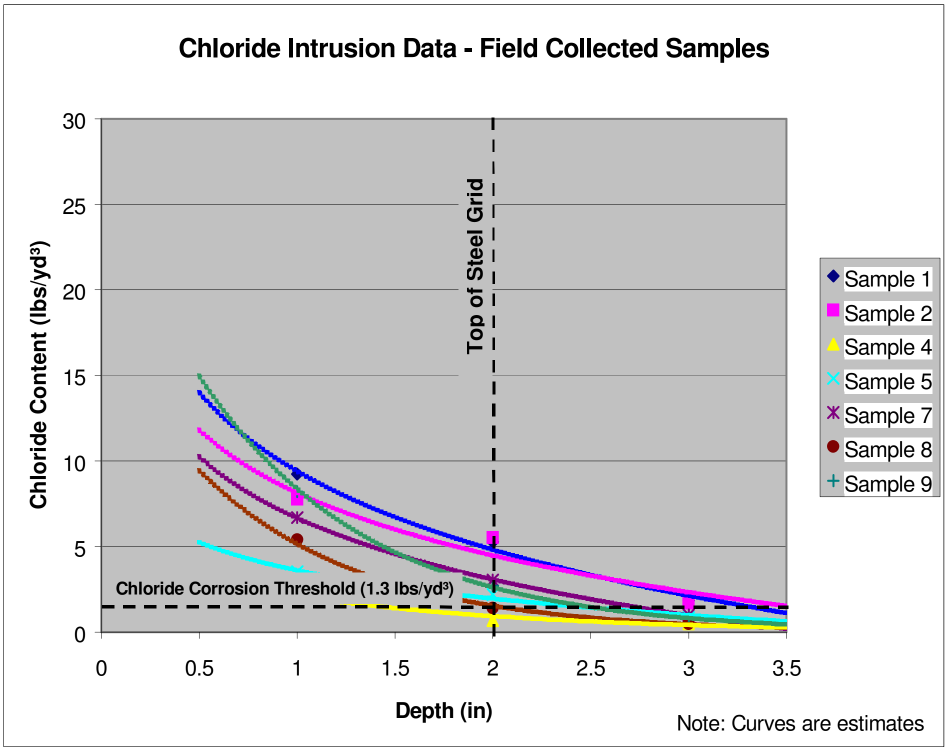
Hole No.	Specimen Depth (in)	Chloride Content (lbs/yd^3)
A2	0.5	10.04
	1	9.12
	1.5	8.84
B1[3]	0.5	7.92
	1	7.32
	1.5	7.92
B2	0.5	7.16
	1	6.12
	1.5	4.76
C1	0.5	14.40
	1	10.12
	1.5	9.72

Note: There is less variation in the concrete chloride content as a function of depth in the lab samples because the differences in depth are less. The letter designation for the lab samples corresponds to the letter assigned to the large permeability cores. Some of the collected field samples were not used, as the information was seen as redundant.

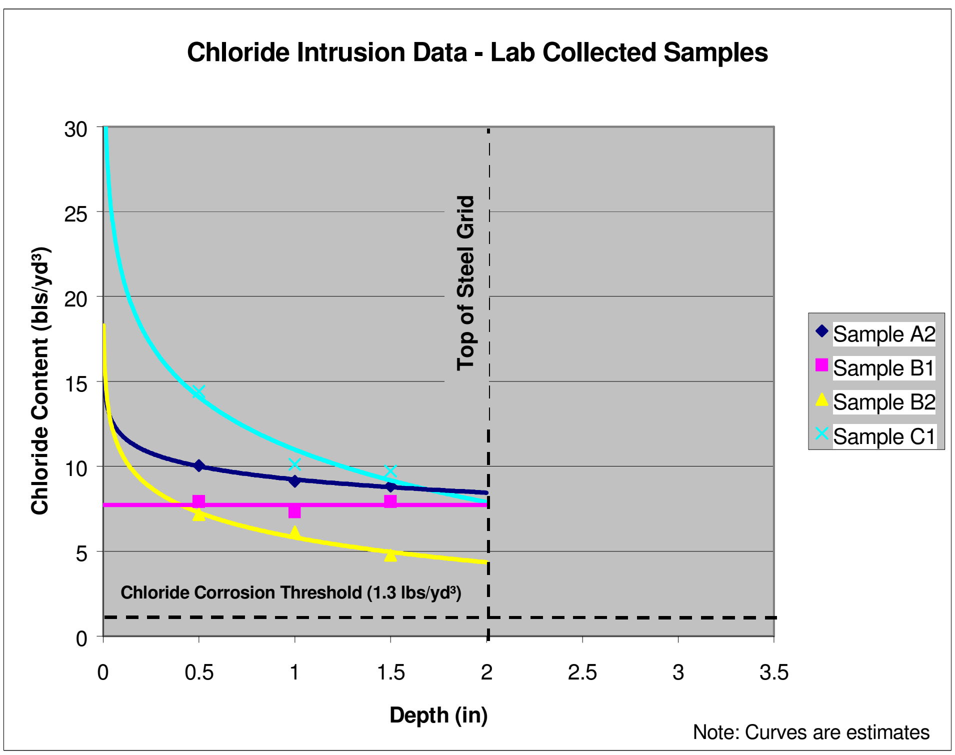
[1]In patched area

[2]In half-cell testing grid

[3]The uniformity of the results for B1 was likely due to a flaw in the concrete such as a crack, although there were none that were visible.



Graph 1 – Chloride Intrusion of Concrete in Field Collected Samples



Graph 2 – Chloride Intrusion of Concrete in Lab Collected Samples

Table 5 – Specific Gravity:

Specimen	Bulk Specific Gravity	Specific Gravity (SSD)	Apparent Specific Gravity	Absorption (% by Wt)
I	2.188	2.220	2.261	1.48
II	2.182	2.219	2.265	1.70
III	2.165	2.199	2.242	1.60
IV	2.175	2.205	2.241	1.36
Avg.	2.178	2.211	2.252	1.535
Density (lbs/yd^3)	136	138	141	-

Note: The specific gravities are low by any measure (typically about 145 lbs/ft^3 for a standard concrete mix). This is due to the high air content and the lack of coarse aggregate.

Table 6 – Rapid Electrochemical Permeability Results:

Core Designation	Core Diameter (in)	Charge (C)	Relative Permeability
A	4	3000	Moderate
B	4	-	-
C	4	4100	High
1B	2.75	4850	High
2	2.75	-	-
3A	2.75	4000	High

Note: Two of the cells malfunctioned during the test and so no results were obtained for cores B and 2. Towards the end of the test, the currents for A and C decreased. This is anomalous behavior; the current normally either continues to increase or reaches a plateau late in the test. There were no indications of problems with these cells and in any case, the actual values, if different, would be higher. As the results are already fairly high, this would have little bearing on the analysis.

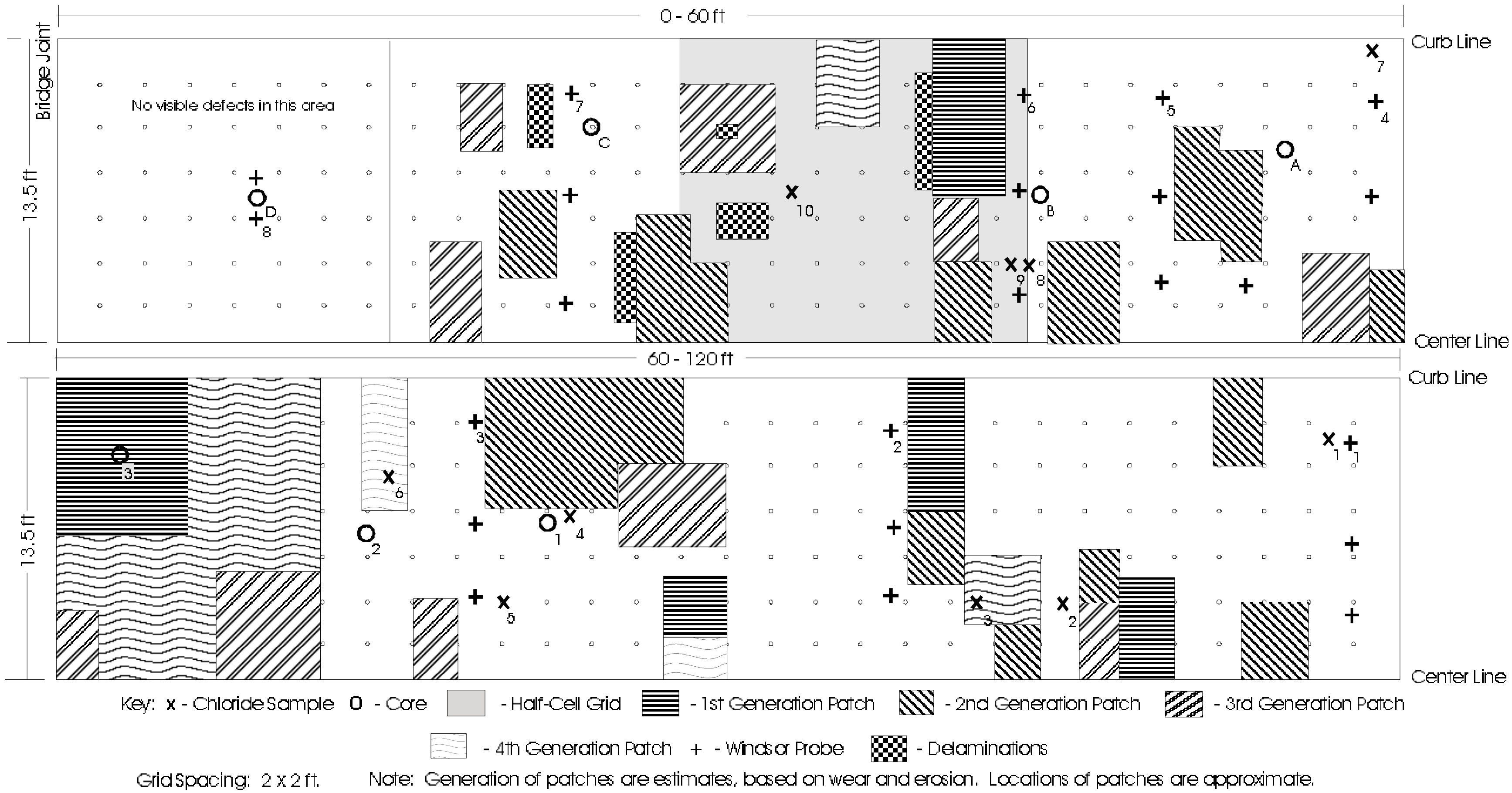


Figure 1 – Map of Defects and Sample Sites in Test Area

ANALYSIS

Delamination Survey:

The deck had relatively few areas of delamination in the examined section (See Fig. 1), a testament to the thoroughness of the ongoing repair operation. Almost certainly, much larger areas would have shown up as delammed, if not for the patching. However, note that of the five delams, two were immediately adjacent to patches and two were in the near vicinity of patches. The use of the jackhammers in removing the existing concrete would tend to cause microcracking of the surrounding concrete when the steel is inevitably struck and the shock wave is sent through the grid. This could ultimately lead to further delamination. The problem is worsened by the nature of the grid system. The grid is a monolithic unit, which means that the shocks will more easily radiate in all directions. This would make the delamination effect more planar, rather than linear as in the case of rebar. The smoothness of the top of the grid also enhances the natural fracture plane.

Concrete Chloride Content:

The chloride values, while much higher than they should be for fourteen year old concrete. At the top of the grid, the values are well above the threshold for the initiation of corrosion (1.3 lbs/yd^3), especially in such a moist environment. That the steel is A588 would only contribute to the problem. This is borne out by the severe signs of corrosion on the concrete removed during patching.

Steel Corrosion:

The half-cell potentials do not indicate any active corrosion, although it was done only in a limited area, because of the confinement in the work zone. Note, however, that there was less evidence of distresses in the area tested and this may have been a better than average section. Testing on a larger scale might or might not indicate corrosion. Since the visual evidence shows that corrosion of the steel has taken place, it is likely that it is continuing.

Electrochemical Indication of Chloride Permeability:

The permeability results are not extremely high, but are significantly above what is typical for concrete of this age and vintage (the high chlorides in the concrete will increase the baseline current in the test, increasing the final result). The size of the smaller specimens did not seem to have a significant effect on the results, as normally, decreasing the diameter of a conductor increases, lowering the charge passed. That the final numbers for 1B and 3A are higher than those for A and C is due to the drop in current through the larger specimens at the end of the test. Until the last third of the test, the currents were tracking together fairly closely, despite the size difference. As significant coarse aggregate normally impedes the flow of current, this tends to bear out the idea of a low percentage of stone in the matrix. Normally, as concrete cures and the cement continues to hydrate, the pore structure closes and becomes less permeable. In this case, either that hasn't happened or more likely, the concrete was very permeable at the start. Cracking in the concrete, not unusual during curing or as a result of loading afterwards, would also increase the permeability. Therefore, the concrete matrix provides only a limited barrier to the intrusion of chlorides presently and originally may have provided little protection for the steel at all.

In-Situ Strength Determination:

The strength of the concrete is relatively low, although given that it is not considered to be structural (from discussions with the consultant), that in itself does not present a problem. This would be consistent with the lack of large coarse aggregate in the mix. The two higher compressive breaks seem more likely to be atypical of the concrete in the deck. This is based on the Windsor Probe results in proximity to the last two compressive cores and the overall pattern of values in the test area. The concrete strengths based on the Windsor Probe data is only an approximation, based on a correlation with the two data points where the cores and probe tests were performed in close proximity. As with any two point comparison, the correlation is only an educated guess. Since the Windsor Probe data is generally used in conjunction with the Moh's number of the coarse aggregate in the mix and there is little to speak of in this concrete, the values in the table provided with the probe were not considered to be reasonable. In Photo 6, note that the failure mode is columnar (ASTM C39, type e). This is more typical of a mortar mix, where there is little aggregate to create shear planes.

Visual Survey:

It is the lack of significant coarse aggregate that may be contributing to the delamination. Whatever the initial cause, once the delamination starts, there is nothing in the concrete matrix to connect the plane of the concrete over the grid to tie it to the concrete within the grid, other than the tensile strength of the concrete itself. The compressive strength is low and typically, the tensile strength is roughly ten percent of the compressive. A tensile strength 200 to 250 psi is not much to resist the forces to which the concrete would be subjected. The top of the grid is a natural fracture plane and since there is no stone that significantly penetrates the plane, there is little to stop the fracture from propagating.

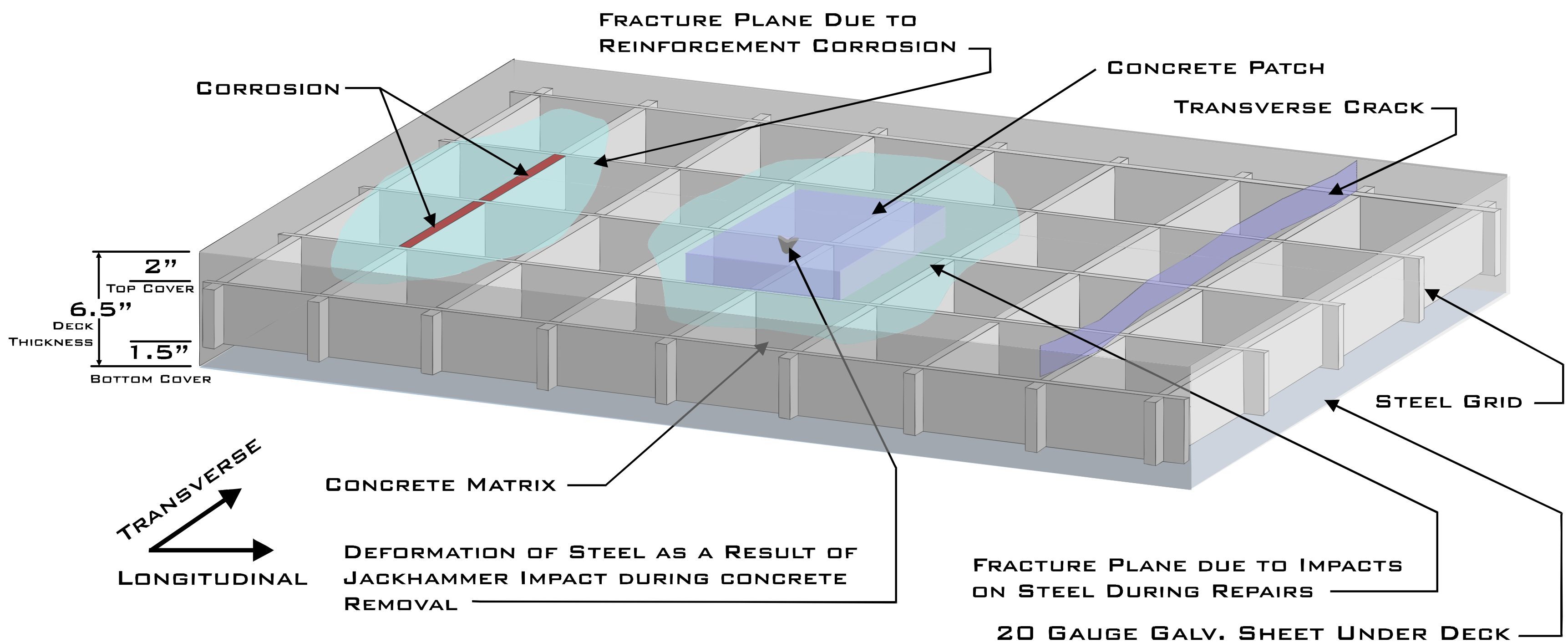


Figure 2 – Delamination Planes and Causes

CONCLUSIONS

There are several problems that are causing the spalling and subsequent delaminations of the wearing surface. Corrosion of the steel is likely the proximate cause of most of the original spalls, although there may be more than one cause for the corrosion. The permeability and cracking of the concrete allows intrusion of chlorides into the matrix; however, it is no worse than other mixes of the era. The wet environment, deicing and sea salts and the weathering steel composition of the grid combine to create corrosion-induced tensile stresses, which initiate spalling on a continual basis. There are also indications of transverse cracking, although we could not identify what caused them or when they originated. These cracks would also significantly increase the ingress of chlorides into the concrete and possibly directly to the steel. There may also be macrocell corrosion taking place, because of the placement of fresh concrete (with low chloride content) in contact with concrete with high chloride levels and the subsequent pH differences created. To counter these problems, a means of reducing the rusting or mitigating its effects will be required.

The methods used in the current repair operations are also contributing to the problem. The specification for the patching needs to be revised. In at least one case, the aggregate used to extend the patching material was dirty and despite the longevity of the repairs, this is bad practice. There is also no quality control or assurance on the operation. Without testing of the material, there is no feedback on how well it is being produced. The workmanship on the repairs is likely the only reason they are lasting as long as they have been. Of paramount concern is the removal process for the existing concrete. The crew has been instructed to go down below the top of the grid to tie the patches into the deck more securely. As noted earlier, this process will cause microcracking in the concrete leading to further delamination. This may be why it is common for several generations of patches to be adjacent to one another. The specification for the material needs to be tightened, QA/QC testing should be required and a less damaging technique for concrete removal needs to be developed.

The lack of any solid tie between the concrete over the grid and that within the grid contributes to the delamination. While there is no direct indication that the dynamics of the bridge directly cause any delams, all suspension bridges sway in the wind, placing decks in unusual loading, including torsion. It is probable that once a delam starts, the bridge dynamics could add to the effect. What should ultimately done about this will depend on what method is selected to repair the deck.

RECOMMENDATIONS

Two options are suggested for repair of the deck. Both involve removing the entire wearing surface and replacing it with an overlay that would be more durable than the existing concrete. Both would also involve treating the steel to slow the corrosion process, although the latter option should be less susceptible to stresses induced as the steel rusts.

The first method is fairly typical and similar to the repair currently being evaluated on the Newport-Pell Bridge. The concrete would be removed down to the about an inch below the top of the grid by Hydro-Demolition. The exposed steel would be cleaned during the removal and coated with an epoxy bonding agent or a broadcast system, primarily to protect it. A new overlay would be placed, consisting of a high performance mix. A corrosion inhibitor would be added to the mix, as an additional measure of protection.

Such a system is proven to be durable and would provide good protection for the steel. Use of a standard HP mix with coarse aggregate would give the connection across the plane of the top of the grid that is lacking now. The use of studs, either welded to the grid or shot into the concrete remaining within the grid openings would also create a more positive connection. The section to be repaired would have to be closed off for several days for the prep work and to allow the placed concrete to reach sufficient strength to bear traffic loads. Given the traffic volume and limited capacity of this two lane bridge, this would be a difficult proposition to put to the motoring public. Concrete mixes are also expensive to place, especially under the conditions existing on the bridge.

If the concrete is in fact not considered structural, the second option is to use a polymer-modified bituminous overlay. As before, Hydro-demolition would be used to remove the surface concrete. The steel would still be cleaned of any visible corrosion in the process. Once, the substrate is exposed, an HP mix would be screeded into the grid openings and brought to the level of the top of the grid. This would differ in placing an HP overlay in that it could be exposed to traffic in about eight hours, because the material would be contained within the grid and less susceptible to damage from traffic. A rubberized chip seal would be placed on this surface, to waterproof and provide a good bond to the substrate. This would be followed by a compatible tack coat and overlaid with a PG 72-34 mix. Such a mix should be highly durable. A maintenance plan would be implemented to monitor the pavement and as necessary, a pavement preservation technique (such as crack sealing and/or microsurfacing) would be employed to extend the life of the overlay. This should be expected in five to seven years after placement.

The long-term performance of these asphalt overlays is not as well known as the high performance concrete, but the process would be substantially faster and less expensive. The flexibility of the overlay would allow it to remain in place longer over corroding steel, although asphalt would provide less early warning of potential problems than a concrete overlay. However, the steel could also be treated prior to placement of the membrane, slowing the corrosion process. Note that in both options, metalizing of the steel would be the preferred option (if possible on weathering steel), but this is an expensive process and the cost to benefit ratio would have to be weighed.

ACKNOWLEDGEMENTS

I would like to thank Ian Frament for his work on this project. As ever, his thoughtful insight into unusual problems is greatly appreciated. His willingness to do what it took to get the job done made this project much easier to manage. I would especially like to acknowledge his suggestion of using a flexible overlay as a solution to the deterioration problem on the deck. Not a typical solution for this situation, but one that may solve a number of problems.

Steven Quintin and Robert Recchia are both to be commended for their efforts. Their assistance was invaluable and their ability to work with minimal supervision allowed me more time to work on other things within a very tight timetable.

Dr. John Walsh performed the testing to determine the chloride contents and as always, did an excellent job, providing reliable results and completing the tests in a timely fashion.

Michael Byrne and José Lima were very helpful in providing their expertise in evaluating the conditions on the bridge and in developing options for addressing the problems raised.

Mr. Lima and Jan Bak reviewed the report and it is better for their perceptive comments.

I would also like to thank the Aetna Bridge repair crew for their help in collecting samples and providing traffic control. Carlos, especially, for his efforts on our behalf.

And finally, to Peter Janaros, thank you for involving me in this project. You bring us such interesting problems!